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River Quality Model for Urban Stormwater Impacts

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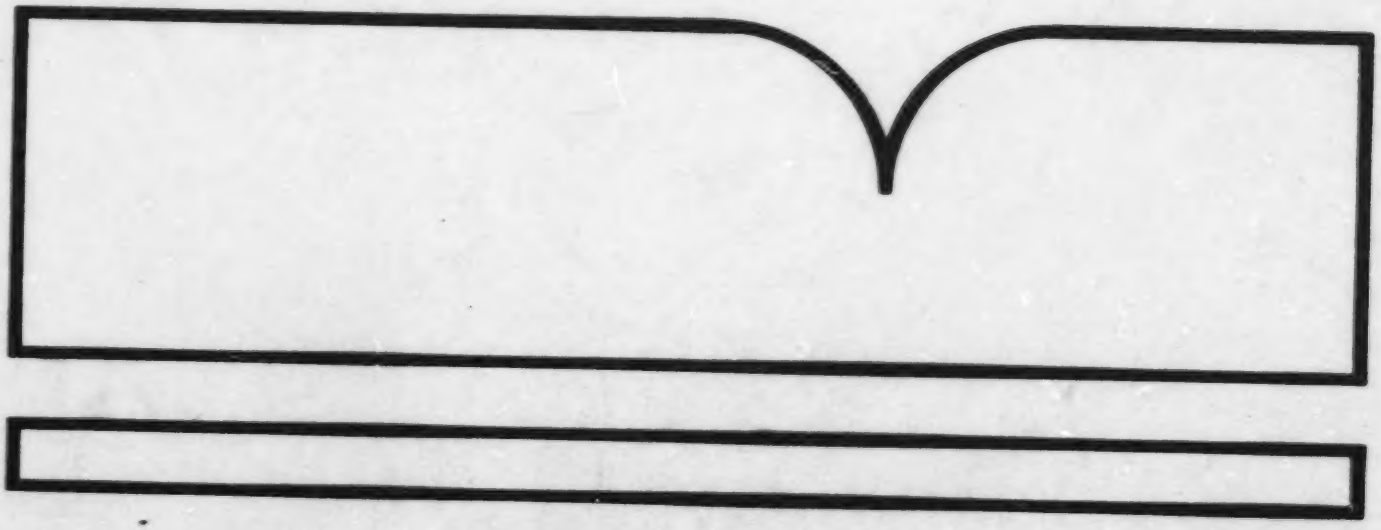

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River Quality Model For Urban Stormwater Impacts

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Introduction

In a 1.67-sq mile (433-ha) urban watershed in Durham, N.C., it was found that the dissolved oxygen (DO) content of the receiving watercourse was independent of the degree of municipal waste treatment beyond secondary treatment during storm flows (2). Approximately one half of the stream miles in the United States are water quality limited and 30% of these stream segments are considered polluted to a certain degree with urban stormwater runoff (4). The implication is that, generally, secondary treatment of dry-weather wastewater flows is insufficient to meet desired receiving water quality standards: therefore, control of runoff pollution must be considered in areawide wastewater management plans and abatement programs. The results of a nationwide assessment of costs and related water quality impacts derived from nonpoint sources (6) were, among others, that:

1. Wet-weather flows represent at least 50% of the total wastewater flow from urban areas.
2. A generalized optimization model, assumning linear costs, predicted primary type facilities are preferable only up to a 10% level of biochemical oxygen

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16. ABSTRACT

A simplified continuous receiving water quality model has been developed as a planning guide to permit preliminary screening of areawide wastewater management strategies. The model simulates the hypothetical response of the stream or tidal river system to the separate and combined effects of waste inputs from upstream sources, dry-weather urban sources and wet-weather urban sources. The total hours of runoff-producing rainfall throughout a year are separated into storm events. For a given storm event, the runoff and pollutant loads are summarized, and critical dissolved oxygen concentrations are estimated as a function of several hydrodynamic and biochemical parameters. An application to the Des Moines River at Des Moines, Iowa, is presented.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Mathematical models, Rivers, Storm runoff, Urban areas, Wastewater treatment, Water pollution control	Combined sewers, Computer programs, Cost effectiveness, Hydrologic models, Storm sewers, Water quality.	13B

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demand (BOD) removal for wet-weather flows, with a secondary type facility preferable for higher levels of control.

3. On a national average basis using BOD removal as the effectiveness parameter, approx 39% of the combined sewer problem and 10% of the other wet-weather flows should be controlled before initiating tertiary treatment of point sources.

The 208 planning effort (Section 208, PL 92-500) established the need for various levels of urban water management analysis (4) to permit preliminary screening of municipal treatment alternatives. Four distinct levels of evaluation techniques, ranging from simple to complex procedures that can be integrated with one another, are summarized in the following:

Level I.—A desktop calculator, statistical analysis procedure, no electronic digital computer required.

University of Florida Methodology—permits the user to estimate the quantity and quality of urban runoff in the combined, storm and unsewered portions of each urban area.

Hydroscience, Inc. Methodology—use of a stormwater simulator and an analytical method based on probability distribution functions and statistical properties of rainfall, runoff, treatment and receiving water impact.

Level II.—A simplified continuous simulation model for planning and preliminary sizing of facilities, developed by Metcalf & Eddy, Inc.; or the computerized optimization version of University of Florida Methodology described in the foregoing.

Level III.—A refined continuous simulation model approach. Continuous hydrologic simulation models, e.g., STORM or continuous SWMM, which generate urban runoff hydrographs and pollutographs are followed by continuous receiving water impact analyses (Level III-Receiving model).

Continuous SWMM—University of Florida
STORM—Corps of Engineers by Water Resources Engineers, Inc.
Level III-Receiving—Duke University

Level IV.—A sophisticated single event simulation model, e.g., EPA SWMM, developed by Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc.

The first three levels essentially represent various degrees of planning detail with models running on hourly time steps for long simulation periods (years). Mathematical complexity and data requirements are kept at a minimum. At the third level, the methods used to generate storm runoff flows and pollutant mass rates or concentrations depend upon the continuous hydrologic simulation model selected by the user. Such techniques are well documented for two models that may be applied to both urban and nonurban watersheds: (1) The Hydrologic Engineering Center's Storage, Treatment, Overflow, Runoff Model (STORM); and (2) the United States Environmental Protection Agency's Storm Water Management Model (SWMM) (8,9). The urban runoff flows and associated pollutant concentrations and mass rates, derived from storm events over the drainage area of interest, represent concurrent time series that are read by Level III-Receiving through the standard card reader devices, or from peripheral storage units (disk/tape/drum). Since the program has the built-in capability of accessing a user-created data set, any continuous hydrologic and water quality model may be considered (15). Detailed single-event models typically use short time steps (minutes) and short simulation times (hours). Their data requirements are usually substantial.

Model Overview

Since it is impractical, if not impossible, to meet stream standards all of the time it appears desirable to express allowable limits for various pollutants on a probabilistic or frequency basis. The approach guiding the development of Level III-Receiving was that the cost-effectiveness of various treatment alternatives can only be determined realistically by a continuous analysis of the frequency of violation of water quality standards. Such an analysis provides an approximation of the integrated system responses to the proposed treatment measures.

The problem of specific interest is to assess the separate and combined effects of the major urban sources of water pollution upon river quality. The relative impact of wastewater sources is appraised by estimating their effect on DO concentrations downstream from the urban area. It is of further interest to distinguish clearly between the relative pollutional impacts of storm sewer discharges and combined sewer overflows. An abstraction of the urban drainage system is presented in Fig. 1. The urban community served by a separate sewer system will convey stormwater runoff and municipal sewage through parallel conduits. The BOD concentration of the storm sewer runoff is mixed with the dry-weather flow (DWF) and accumulated sewer solids. An interceptor carries the sanitary design flow to the municipal sewage treatment plant. The combined sewer overflow is either given treatment or allowed to discharge directly to the receiving water. Since complete mixing is assumed, the BOD concentrations of the combined sewer overflow Q_o and the flow (DWFCMB) intercepted for treatment by the DWF facility are identical. Any desired degree of treatment may be imposed at both the DWF and the wet-weather flow (WWF) treatment plants. The concentration of the combined BOD inputs in the receiving water is given by

$$BOD_m = \frac{BOD_u Q_u + BOD_d Q_d + BOD_w Q_w}{Q_u + Q_d + Q_w} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which BOD_m = mixed BOD concentration in receiving water, in milligrams per liter; BOD_u = mixed BOD concentration from sources upstream of urban area, in milligrams per liter; BOD_d = BOD concentration of dry-weather flow treatment plant effluent, in milligrams per liter; BOD_w = BOD concentration of wet-weather flow treatment facility effluent, in milligrams per liter; Q_u = upstream flow, in cubic feet per second; Q_d = DWF treated effluent, in cubic

feet per second; and Q_w = WWF treated effluent, in cubic feet per second.

The BOD concentrations of the DWF and WWF treated effluents are given by:

$$BOD_d = \frac{[(BOD_f DWFSEP) + (BOD_c DWFCMB)](1 - R_d)}{DWFSEP + DWFCMB} \quad \ldots\ldots\ldots (2)$$

$$BOD_w = \frac{[(BOD_s Q_s) + (BOD_c Q_c)](1 - R_w)}{Q_s + Q_c} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

in which BOD_f = BOD concentration of municipal sewage, in milligrams per

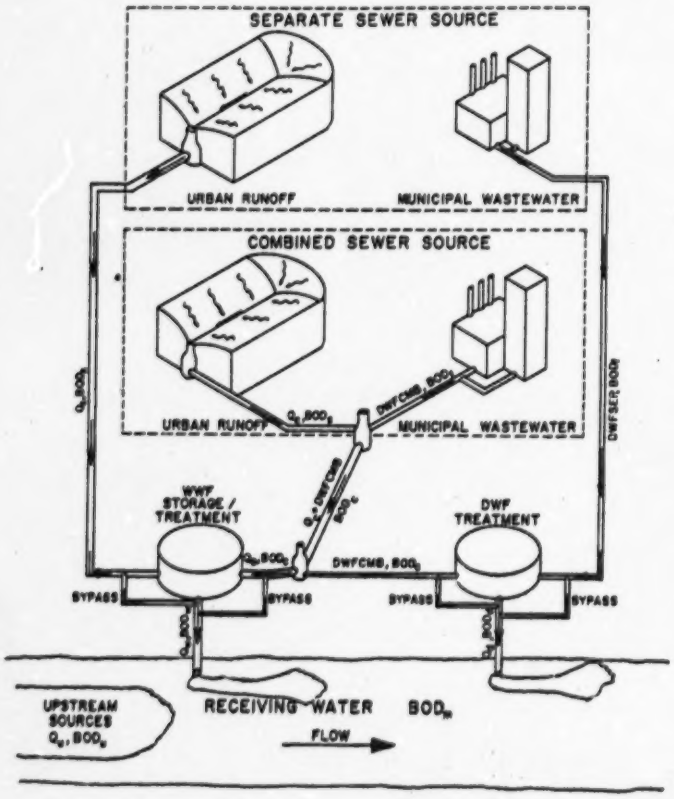

FIG. 1.—Abstraction of the Urban Drainage System

liter; BOD_c = mixed BOD concentration in the combined sewer, in milligrams per liter; BOD_s = BOD concentration of urban stormwater runoff, in milligrams per liter; DWFSEP = DWF contribution from separate sewer area, in cubic feet per second; DWFCMB = DWF contribution from combined sewer area,

in cubic feet per second; Q_s = urban runoff carried by the separate storm sewer, in cubic feet per second; Q_c = combined sewer flow, in cubic feet per second; R_d = fraction removal of BOD achieved by the DWF treatment facility, and R_w = fraction removal of BOD achieved by the WWF treatment facility.

The initial conditions of BOD in the river are defined by Eq. 1, and the hypothetical impact on the oxygen balance of the receiving stream is estimated by using simplified mathematical modeling approaches. The total hours of runoff-producing rainfall throughout the year are separated into storm events by defining a minimum interevent time. For a given storm event, the runoff and pollutant loads are summed and the critical DO deficit is estimated as a function of several stream parameters: (1) Temperature; (2) flow, (3) oxygen concentration; (4) deoxygenation and reaeration rates; (5) longitudinal dispersion; and (6) BOD concentrations. The minimum DO is calculated subsequently and a frequency analysis is performed. Stream velocity is computed as a function of the discharge and the time and distance to each critical deficit point are obtained for each event.

The options used for the simulations include:

1. Five inflow combinations of river flow plus: (a) DWF; (b) DWF + separate flow; (c) DWF + combined flow; (d) separate flow + combined flow; and (e) separate flow + combined flow; and (e) separate flow + combined flow + DWF.
2. Four DWF treatment rates (variable).
3. Three WWF treatment rates (variable).
4. Three fractions of measured upstream flow.

Item 4 is included as a model option to investigate whether the relative impact of urban stormwater runoff is significant in the upstream portions of river basins. This effect may be simulated by simply reducing the upstream flow to any desired fraction of its actual measured value. Thus, discharge into a dry river bed may be studied.

The impact of urban runoff on the quality of the receiving stream is evaluated in terms of: (1) Critical DO frequency curves, i.e., by the percent of time predicted minimum DO levels fall below a specified concentration; and (2) classical DO sag curves for each inflow combination. During periods of wet weather, the program performs computations on an hourly time step. Otherwise, a daily time step is used. An economic evaluation of various control strategies augments the decision-making capabilities of such an approach.

Event Definition

The basic approach to define a wet-weather event is to analyze the runoff time series and establish the minimum number of consecutive dry-weather hours (DWH) that separates independent storm events. The independence of these events is not defined in a strictly climatologic sense. The DWH refer to periods during which no runoff was produced.

The runoff time series is subjected to autocorrelation analysis. For hydrologic processes, it is practical to estimate the autocorrelation coefficients by an

open-series approach (5,18). Tolerance limits for a normal random time series which is circular and of lag 1 have been estimated for any percent probability level (1), and the technique has been extended for use with an open series for the general lag case (18). A plot of the serial correlation coefficients, $r(k)$, against the number of lags, k, is called a correlogram. Autocorrelation analysis is essentially a study of the behavior of the correlogram of the process under investigation (16). The model compares the value of $r(k)$ with the tolerance limit at the corresponding number of hourly lags, k. The *minimum interevent time* (MIT) which separates independent wet-weather events is defined as the minimum value of k, for which $r(k)$, is not significantly different from zero at a 95% probability level (15).

Effect on Receiving Waters

A simplified mathematical modeling approach is used in which critical deficits and resulting minimum DO concentrations are determined for a large number of waste input combinations, treatment schemes, and receiving water conditions. The development of a detailed and sophisticated model is not justified for the problem context: to provide adequate information on the relative effectiveness of various pollutant control strategies in achieving selected water quality standards. The basic theory of mathematical modeling of one-dimensional bodies of water is applied for the spectrum of natural systems from freshwater streams to tidal rivers. The approach is particularly advantageous for a limited data base on natural system geometry, hydrodynamic variables, and water. Assumptions typical of models limited for preliminary planning are made (10):

1. Temporal steady-state conditions prevail, in which all system parameters and inputs are constant with respect to time, however, a relatively short time step, 1 h, is used for simulation.
2. Natural system parameters (such as flow, velocity, hydraulic depth, deoxygenation and reaeration rates, and longitudinal dispersion) are spatially constant along the flow axis throughout each time step.
3. All waste inflows to the receiving body of water occur at one point.
4. The effects of various natural biological processes (algal photosynthesis and respiration, benthal stabilization) are incorporated into a background quality that is reflected by DO deficit (if none, by saturation) upstream from the waste inflow point. Any benthic buildup is incorporated into the BOD decay rate.
5. Waste treatment facilities operate at constant efficiencies, independent of hydraulic and organic loadings, for the entire period of simulation.

Some additional observations are appropriate with respect to the validity of simulating a transient situation with a steady state model such as Level III-Receiving:

1. It essentially averages pollutant loads and receiving stream conditions over an event duration, using an hourly time step.
2. It assumes that increased wet-weather flow will not cause sudden scour of benthal deposits sufficient to drive the DO to zero.
3. It assumes the time scale of the urban runoff event is short compared to the time scale of the receiving stream response, i.e., time to critical DO deficit.

When the latter condition holds, transient pulses are greatly damped. Thus, the model is not suitable for use on small, *flashy* streams where most of the hydrograph consists of urban runoff. Level III-Receiving uses DO, because in spite of limitations on its implications on receiving water quality, it still represents the only parameter, besides bacteria, defined as a water quality *standard* in most states. For example, there are seldom any nutrient, sediment or toxic standards. Furthermore, these standards apply to dry-weather periods only. None of the regulatory agencies have established an *allowable frequency of violation* of water quality standards.

Oxygen Balance of Polluted Streams and Tidal Rivers.—In view of the modeling objectives, pollutant transport processes in these systems are approximated by the steady-state, one-dimensional version of the classical convective diffusion equation:

$$0 = \frac{\partial}{\partial x}\left[E\frac{\partial C}{\partial x} - UC\right] \pm \Sigma S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which C = concentration of water quality parameter (pollutant), M/L^3; $-E(\partial C/\partial x)$ = mass flux due to longitudinal dispersion along the flow axis, the x direction, M/L^2T; UC = mass flux due to advection by the fluid containing the mass of pollutant, M/L^2T; S = sources or sinks of the substance C, M/L^3T; U = flow velocity, L/T; and E = longitudinal dispersion coefficient, L^2/T.

The equation assumes no diffusion of pollutants through the body of water boundaries (other than what may be included in the source-sink term) and is best suited to predict concentrations relatively far downstream from the point of waste injection. Since critical DO deficits usually occur some distance downstream from the waste source, Eq. 4 is adequate for such predictions. The main sources of dissolved oxygen in stream systems are *atmospheric reaeration* and oxygen production by *photosynthesis*. The major *sinks* include *carbonaceous oxygen demand* (CBOD), *nitrogenous oxygen demand* (NBOD), *benthal demand*, and respiration of aquatic plants. In the model, the effects of photosynthesis, respiration, and benthal demand are assumed to be incorporated into the measured upstream DO deficit. Field measurements of organic and ammonia nitrogen present in all wastewater inputs to receiving waters are seldom available, much less during runoff events. Thus, nitrogenous oxygen demand is currently neglected. Since it is desired to solve for the DO deficit and

$$\frac{\partial C}{\partial x} = -\frac{\partial D}{\partial x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Eq. 4 becomes:

$$E\frac{d^2D}{dx^2} - U\frac{dD}{dx} + K_1L_0e^{mx} - K_2D = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which $m = (U/2E)(1 - \sqrt{1 + [(4K_1E)/U^2]})$ for $x \geq 0$; D = DO deficit, in milligrams per liter; E = longitudinal dispersion coefficient, in square feet

per hour; U = freshwater stream or tidal river velocity, in feet per hour; K_2 = atmospheric reaeration coefficient, per hour; K_1 = deoxygenation constant of carbonaceous BOD, per hour; and L_0 = remaining carbonaceous BOD concentration at $x = 0$, in milligrams per liter.

Critical Deficit and DO Levels.—The solution to Eq. 6 as a function of time since release is given by

$$D = \frac{L_0 K_1}{K_2 - K_1}\left(e^{jt} - \frac{s_1}{s_2}e^{gt}\right) + D_0 e^{gt} \quad \ldots\ldots (7)$$

in which D_0 = initial DO deficit, in milligrams per liter; $t = x/U$ = translational time, in hours; $j = [U^2/(2E)]\{1 - \sqrt{1 + [(4K_1E)/U^2]}\}$, per hour; $g = [U^2/(2E)]\{1 - \sqrt{1 + [(4K_2E)/U^2]}\}$, per hour; $s_1 = \sqrt{1 + [(4K_1E)/U^2]}$, dimensionless; and $s_2 = \sqrt{1 + [(4K_2E)/U^2]}$, dimensionless.

The time at which the maximum deficit occurs is obtained by taking the partial derivative of Eq. 7 with respect to time, setting the resulting equation to zero and solving for time. After simplification and convenient substitutions

$$t_c = \frac{1}{(j - g)}\ln\left[\frac{g}{j}\left(\frac{s_1}{s_2} - fR_0 + R_0\right)\right] \quad \ldots\ldots (8)$$

in which t_c = elapsed time at which the critical deficit occurs, in hours; f = self-purification ratio = K_2/K_1; and R_0 = ratio of the initial DO deficit, D_0, to the initial BOD, L_0, dimensionless.

Finally, the critical deficit is found by substituting the value of t_c given by Eq. 8 into Eq. 7:

$$D_c = \frac{L_0 K_1}{K_2 - K_1}\left[e^{jt_c} - \frac{s_1}{s_2}e^{gt_c}\right] + D_0 e^{gt_c} \quad \ldots\ldots (9)$$

in which D_c = critical (maximum) deficit, in milligrams per liter.

The minimum DO level is calculated as

$$C_{min} = C_s - D_c \quad \ldots\ldots (10)$$

in which C_{min} = concentration of DO at maximum deficit, in milligrams per liter; and C_s = saturation concentration of DO, in milligrams per liter. The saturation concentration is determined from the regression relationship (17)

$$C_s = 14.652 - 0.41022\,T + 0.0079910\,T^2 - 0.000077774\,T^3 \quad \ldots\ldots (11)$$

in which T = water temperature, in degrees Celsius.

For freshwater streams, the advective flux is significantly larger than the mass flux due to longitudinal dispersion. In a tidal river, the advective and dispersive fluxes are both significant. However, it should be noted that even for large, deep rivers, the effect of dispersion generally cannot be neglected when the wastewater inputs are time variable (13,14). If the model user wishes to neglect the dispersive flux, the classical Streeter-Phelps formulations are then applied.

The Langbein and Durum (11) equation for the prediction of the reaeration coefficient was chosen because it is most closely related to subsequent procedures applied to obtain stream velocity and depth:

$$K_2 = 2.303\left[3.3\frac{U}{H^{1.33}}\right] \quad \ldots\ldots (12)$$

in which K_2 = reaeration coefficient at 20° C, per day; U = stream velocity, in feet per second; and H = stream depth, in feet.

The problem lies in obtaining values of U and H, since the streamflow varies with time. In the absence of measurements, or if the data cannot be obtained in an expedient manner (as in the ensuing application to the Des Moines River), an approximation can be made (12) which uses strong correlations between velocity versus flow and depth versus flow, namely:

$$U = \alpha_1 Q^{\alpha_2} \quad \ldots\ldots (13)$$

$$H = \beta_1 Q^{\beta_2} \quad \ldots\ldots (14)$$

in which Q = streamflow, in cubic feet per second; and α_1, α_2, β_1, β_2 = regression coefficients. The model user also has the option of computing the deoxygenation coefficient, K_1, as a power function of water depth (10):

$$K_1 = \gamma_1 H^{\gamma_2} \quad \ldots\ldots (15)$$

in which γ_1, γ_2 = regression coefficients. Temperature corrections are applied to carbonaceous BOD, deoxygenation and reaeration rate coefficients.

Application to Study Area

The city of Des Moines, Iowa is located near the confluence of the Des Moines River and the Raccoon River. It contains approx 200,000 out of the total population of 288,000 for the metropolitan area (3). The mean annual precipitation is 31.27 in. (795 mm) which is approximately equal to the United States average, and the average value for the state of Iowa. The urban area covers 49,000 acres (19,830 ha) of land which has gently rolling terrain. Most of the area, 45,000 acres (18,211 ha), is served by separate sewers, while 4,000 acres (1,619 ha) are served by combined sewers. Davis and Borchardt (3) conducted an extensive sampling program of combined sewer overflows, stormwater discharges, and surface waters in the metropolitan Des Moines area from March 1968–October 1969.

The autocorrelation function of hourly runoff events for Des Moines, Iowa is presented in Fig. 2. An essentially zero correlation (within 95% tolerance limits) between runoff events first occurs at a lag of approx 9 h, defining the minimum interevent time. The physical interpretation is that periods without runoff for at least 9 h separate uncorrelated, and therefore independent, wet-weather events. The hourly urban runoff and associated pollutant loads within each event (including DWF pollutant loads during DWH periods less than 9 h duration) are summed, average conditions are determined, and the model proceeds with the receiving water analysis.

Model calibration was preceded by calibration of the urban runoff BOD, loading rates for Des Moines, Iowa as computed by STORM. Model parameters were adjusted to obtain a fit between calculated and observed profiles of DO (*standard error of estimate* = 1.86 mg/L). These curves are shown in Fig. 3, along with other pertinent information, and correspond to a point 5.6 miles

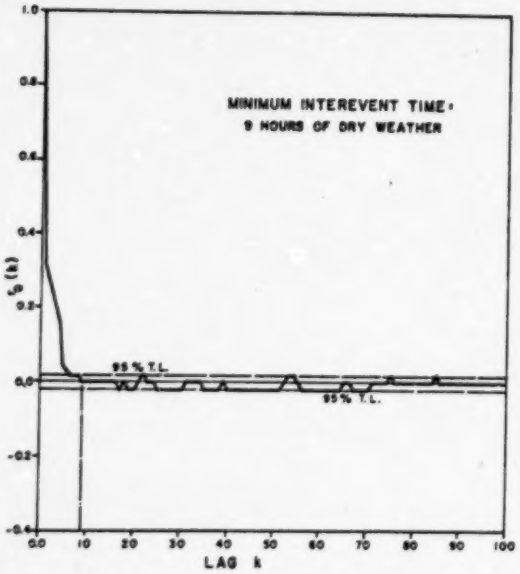

FIG. 2.—Autocorrelation Functions of Hourly Urban Runoff for Des Moines, Iowa, 1968

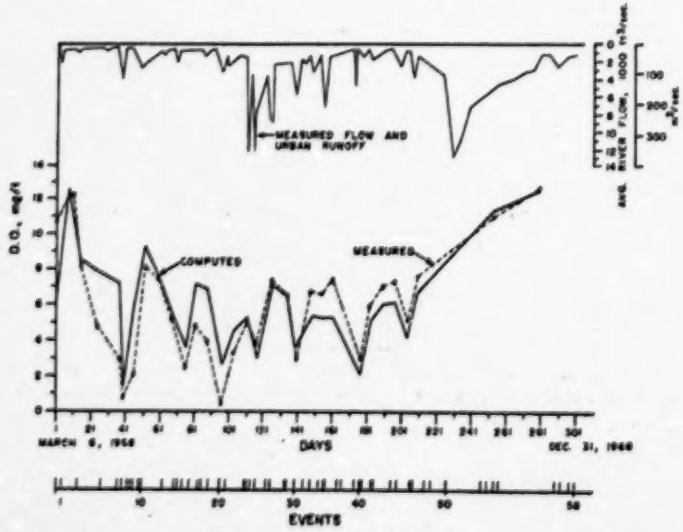

FIG. 3.—Application to Des Moines, Iowa. Measured and computed values of DO at 5.6 miles (9.0 km) downstream from confluence of Raccoon and Des Moines Rivers

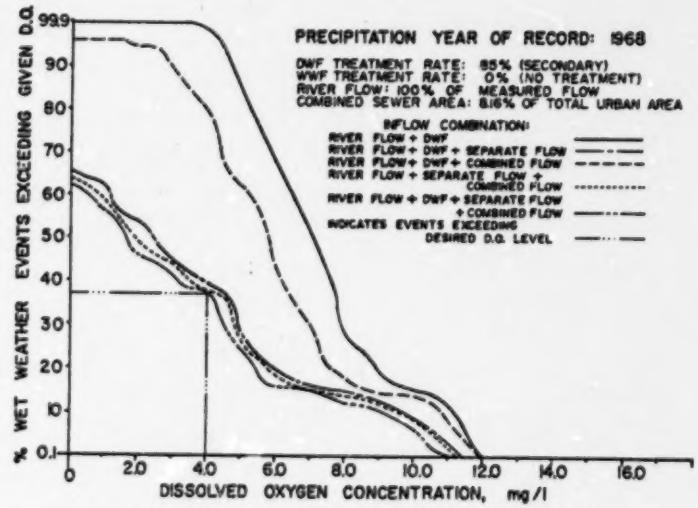
PRECIPITATION YEAR OF RECORD: 1968
DWF TREATMENT RATE: 85% (SECONDARY)
WWF TREATMENT RATE: 0% (NO TREATMENT)
RIVER FLOW: 100% OF MEASURED FLOW
COMBINED SEWER AREA: 8.16% OF TOTAL URBAN AREA
INFLOW COMBINATION:
RIVER FLOW + DWF
RIVER FLOW + DWF + SEPARATE FLOW
RIVER FLOW + DWF + COMBINED FLOW
RIVER FLOW + SEPARATE FLOW + COMBINED FLOW
RIVER FLOW + DWF + SEPARATE FLOW + COMBINED FLOW
INDICATES EVENTS EXCEEDING DESIRED D.O. LEVEL
% WET WEATHER EVENTS EXCEEDING GIVEN D.O.
DISSOLVED OXYGEN CONCENTRATION, mg/l

FIG. 4.—Minimum DO Frequency Curves for Existing Conditions in the Des Moines River

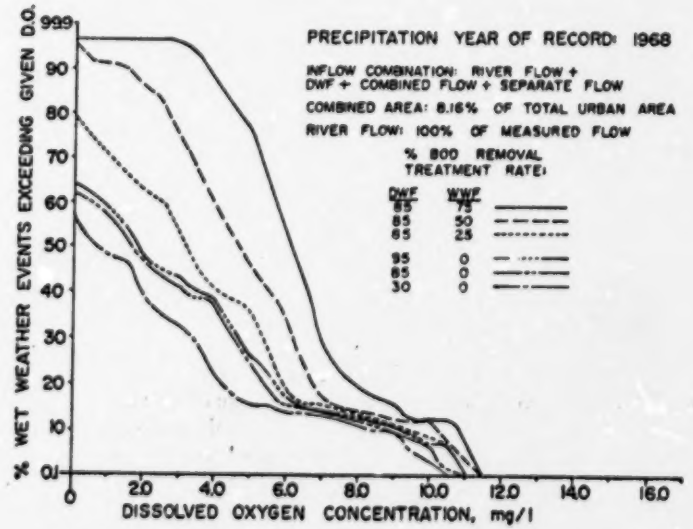
PRECIPITATION YEAR OF RECORD: 1968
INFLOW COMBINATION: RIVER FLOW + DWF + COMBINED FLOW + SEPARATE FLOW
COMBINED AREA: 8.16% OF TOTAL URBAN AREA
RIVER FLOW: 100% OF MEASURED FLOW
% BOD REMOVAL TREATMENT RATE:
DWF WWF
85 75
85 50
85 25
95 0
85 0
30 0
% WET WEATHER EVENTS EXCEEDING GIVEN D.O.
DISSOLVED OXYGEN CONCENTRATION, mg/l

FIG. 5.—Minimum DO Frequency Curves for Various Treatment Alternatives

(9.0 km) downstream from the confluence of the Raccoon and Des Moines Rivers. The average time to maximum DO deficit in the stream was 32 h (about 40 miles downstream); and the average wet-weather event duration was 6 h (minimum = 1 h, maximum = 24 h), as defined by a minimum interevent time of 9 DWH. Thus, the assumption that the time scale of the urban runoff event is short when compared to receiving water response time is reasonable.

The total rainfall that fell over Des Moines, Iowa during 1968 was 27.59 in. (701 mm): STORM computed a total runoff of 10.28 in. (261 mm). A large number of wastewater inflow and treatment rate combinations were investigated and the complete results are available elsewhere (15). It is appropriate to examine the model estimates of critical DO concentration in the Des Moines River, for all waste inputs, for conditions assumed to exist in 1968 during periods of urban runoff: (1) Secondary treatment (85% BOD removal) of DWF; (2)

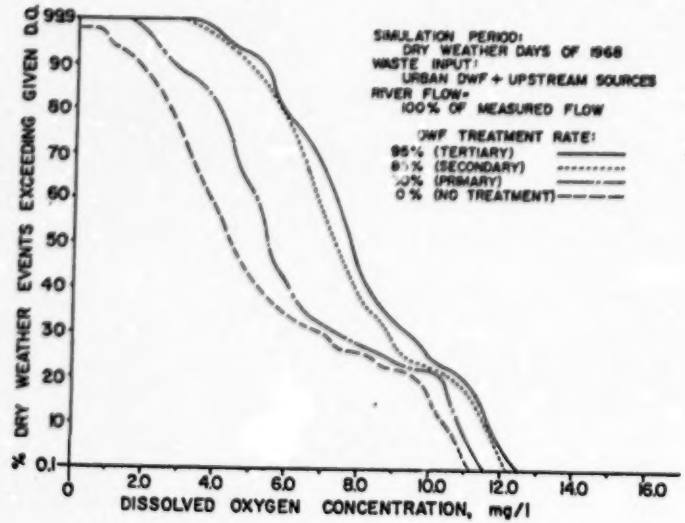

FIG. 6.—Dry-Weather DO Frequency Curves for Various DWF Treatment Alternatives

no stormwater treatment; (3) full river flow (100% of measured flow); and (4) combined sewer area equal to 8.16% of the total urban area. Fig. 4 represents the minimum DO frequency curves for these conditions. The curves clearly indicate that all combinations including a substantial amount of wet-weather flow (WWF) result in a drastic decrease in river minimum DO concentrations. For example, 63% of all wet-weather events caused minimum DO levels below 4.0 mg/L. Raw sewage and primary effluent were by-passed to the Des Moines River on an intermittent basis from January–August (accounted for in the simulation) due to construction of high-rate trickling filters.

The minimum DO frequency curves in Fig. 5 compare six treatment alternatives to reduce water pollution during periods of urban runoff. Even option three, with a small percentage of urban runoff treatment, is superior to all alternatives

controlling DWF only. For the dry-weather simulation period, upstream river flow was on the average 94% of total river flow, ranging from 82%–99.6%. The results are shown in Fig. 6. Ninety-seven percent of the dry-weather days exceed a minimum DO concentration of 4.0 mg/L with DWF secondary treatment.

Cost-Effectiveness of Control Alternatives

Six alternatives to existing DWF treatment in Des Moines during 1968 are considered in the evaluation:

1. Upgrading the treatment plant from a high-rate trickling filter configuration to an activated sludge-coagulation-filtration system (85% BOD removal to 95%).
2. Downgrading DWF treatment to primary (85% BOD removal to 30%).
3. Providing urban runoff control at four levels of BOD removal while maintaining secondary treatment of DWF: (a) 25%; (b) 50%; (c) 75%; or (d) 85% WWF treatment.

To achieve a tertiary treatment configuration, the Des Moines DWF plant must undergo an intermediate-stage modification to activated sludge. A detailed cost analysis is presented in the nationwide assessment (6). Cost estimates of providing various storage/treatment mixes for stormwater control are based on the results of a constrained optimization model (7). Total annual cost for the optimal solution can be approximated by functions of the form:

$$Z^* = ke^{\beta R_l} \quad \ldots\ldots\ldots\

TABLE 1.—Annual Costs for Control Alternatives

Alternative (1)	Total annual cost,[a] in dollars per year (2)
Dry weather[b]	
activated sludge-coagulation-filtration	3,507,000
high-rate trickling filter	1,843,000
primary	405,000
Wet-weather control,[c] (BOD removal as a percentage)	
25	816,000
50	2,682,000
75	9,293,000
85	15,479,000

[a]Includes amortized capital costs (20 yr, 8%, Engineering News-Record index 2200), operating and maintenance costs.

[b]For plant size 35.3 mgd (1.55 m^3/s).

[c]Based on 45,000 acres (18,211 ha) of storm sewer area; 4,000 acres (1,619 ha) of combined sewer area.

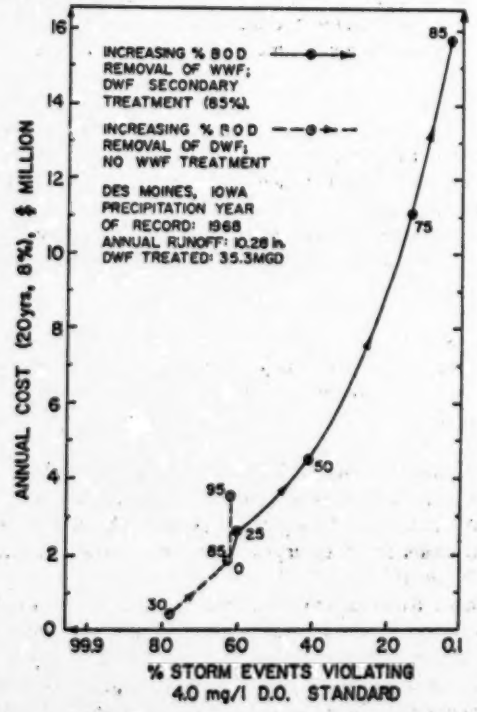

FIG. 7.—Cost Effectiveness of Control Alternatives during Periods of Urban Runoff

should be considered only after a secondary treatment technology of DWF has been achieved. However, beyond that level 25% and 50% WWF treatment become competitive with DWF tertiary treatment technology. It is clearly shown in Fig. 8 that, when the entire period of simulation (1 yr) is considered, 85% WWF treatment becomes asymptotic to the minimum number of violations that can be achieved, at a very high cost. Both 25% and 50% WWF treatment

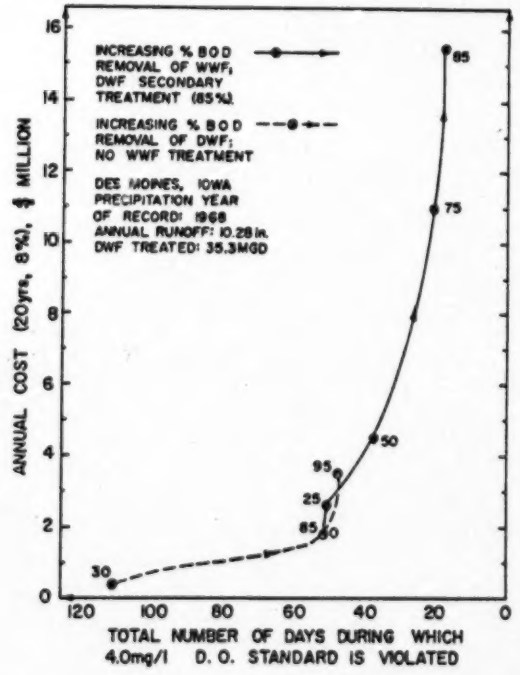

FIG. 8.—Cost Effectiveness of Control Alternatives during Entire Simulation

remain strong alternatives to higher than secondary treatment of DWF in relative cost-effectiveness.

Summary and Conclusions

A simplified river quality model has been developed to permit preliminary planning and screening of areawide urban wastewater treatment alternatives, in terms of frequency of violations and more traditional approaches such as DO profiles. It evaluates the stream response to the separate and combined effects of waste inputs from: (1) Upstream sources; (2) dry-weather urban sources; and (3) wet-weather urban sources. Among its capabilities are:

1. Level III-Receiving may be interfaced through peripheral storage devices

with various hourly, continuous urban catchment hydrologic simulation models.

2. A large number of wastewater inflow combinations, dry-weather flow and wet-weather flow treatment rates, and upstream flow conditions may be simulated. Thus, a comparative evaluation of control alternatives is possible in terms of their subsequent impact on receiving water quality.

3. Continuous analysis allows representation of water quality impacts due to the random occurrence and probabilistic nature of hydrologic phenomena.

4. It computes a minimum interevent time to define statistically independent storm events.

Level III-Receiving has been developed on a general basis so that it may be applied to the surface drainage phase of most urban catchments by simply changing the input data to reflect the particular study area and hydrologic time series. There is virtually no limitation to the size of catchment modeled. In theory, an unlimited number of storm events may be processed; however, practical considerations such as computer time and costs may be limiting to some users. Data requirements are common to engineering analysis of nonpoint source problems. Field measurements, quantitative and qualitative, are necessary to adequately calibrate model parameters and verify predicted values. An application to Des Moines, Iowa has been presented. Results indicate that wet-weather control should be considered after a secondary treatment technology has been achieved to control dry-weather wastewater inputs.

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Appendix I.—References

1. Anderson, R. L., "Distribution of the Serial Correlation Coefficients," *Annals of Mathematical Statistics*, Vol. 13, 1942, pp. 1–13.
2. Colston, N. V., "Characterization and Treatment of Urban Land Runoff," *Technical Report EPA-670/2-74-096*, United States Environmental Protection Agency, Washington, D.C., Dec., 1974.
3. Davis, P. L., and Borchardt, F., "Combined Sewer Overflow Abatement Plan," *Technical Report EPA-R-73-170*, United States Environmental Protection Agency, Washington, D.C., Apr., 1974.
4. Field, R., Tafuri, A. N., and Masters, H. E., "Urban Runoff Pollution Control Technology Overview," *Technical Report EPA-600/2-77-047*, United States Environmental Protection Agency, Washington, D.C., Mar., 1977.
5. Fiering, M. B., and Jackson, B. B., "Synthetic Streamflows," *Water Resources Monograph* 1, American Geophysical Union, Washington, D.C., 1971.
6. Heaney, J. P., et al., "Nationwide Evaluation of Combined Sewer Overflows and Urban Stormwater Discharges, Volume II: Cost Assessment and Impacts," *Technical Report EPA-600/2-77-0646*, United States Environmental Protection Agency, Washington, D.C., Mar., 1977.
7. Heaney, J. P., Nix, S. J., and Murphy, M. P., "Storage-Treatment Mixes for Stormwater Control," *Journal of the Environmental Engineering Division*, ASCE, Vol. 104, No. EE4, Proc. Paper 13966, Aug., 1978, pp. 581–592.
8. Huber, W. C., et al., "Interim Documentation, November 1977 Release of EPA SWMM," University of Florida, Gainesville, Fla., 1977.
9. "Urban Stormwater Runoff: STORM," *Generalized Computer Program 723-S8-L7520*, Hydrologic Engineering Center, Corps of Engineers, July, 1976.
10. "Simplified Mathematical Modeling of Water Quality," Hydroscience, Inc., United States Environmental Protection Agency, Washington, D.C., Mar., 1971.
11. Langbein, W. B., and Durum, W. H., "The Aeration Capacity of Streams," *United States Geological Survey Circular 542*, Washington, D.C., 1967.
12. Leopold, L. B., and Maddock, T., "The Hydraulic Geometry of Stream Channels and Some Physiographic Implications," *United States Geological Survey Professional Paper 252*, Washington, D.C., 1953.
13. Liu, H., "Predicting Dispersion Coefficient of Streams," *Journal of the Environmental Engineering Division*, ASCE, Vol. 103, No. EE1, Proc. Paper 12724, Feb., 1977, pp. 59–69.
14. McQuivey, R. S., and Keefer, T. N., "Simple Method for Predicting Dispersion in Streams," *Journal of the Environmental Engineering Division*, ASCE, Vol. 100, No. EE4, Proc. Paper 10708, Aug., 1974, pp. 997–1011.
15. Medina, M. A., "Level III: Receiving Water Quality Modeling for Urban Stormwater Management," *Technical Report EPA-600/2-79-100*, United States Environmental Protection Agency, Washington, D.C., Aug., 1979.
16. Quimpo, R. G., "Autocorrelation and Spectral Analyses in Hydrology," *Journal of the Hydraulics Division*, ASCE, Vol. 94, No. HY2, Proc. Paper 5837, Mar., 1968, pp. 363–373.
17. "Solubility of Atmospheric Oxygen in Water," by the Committee on Sanitary Engineering Research, N. L. Nemerow, Chmn., *Journal of the Sanitary Engineering Division*, ASCE, Vol. 86, No. SA4, Proc. Paper 2556, July, 1960, pp. 41–53.
18. Yevjevich, V., "Stochastic Processes in Hydrology," Water Resources Publications, Fort Collins, Colo., 1972.

Appendix II.—Notation

The following symbols are used in this paper:

- BOD = biochemical oxygen demand, in milligrams per liter;
- BOD_c = mixed BOD concentration in the combined sewer, in milligrams per liter;
- BOD_d = BOD concentration of dry-weather flow treatment facility effluent, in milligrams per liter;
- BOD_f = BOD concentration of municipal sewage, in milligrams per liter;
- BOD_m = mixed BOD concentration in receiving water, in milligrams per liter;
- BOD_s = BOD concentration of urban stormwater runoff, in milligrams per liter;
- BOD_u = mixed BOD concentration from sources upstream of urban area, in milligrams per liter;
- BOD_w = BOD concentration of treated wet-weather effluent, in milligrams per liter;
- BOD_5 = standard BOD test, 5 days at 68° F (20° C), in milligrams per liter;
- C = concentration of water quality parameter, M/L^3;
- C_{min} = concentration of DO at maximum deficit, in milligrams per liter;
- C_s = saturation concentration of DO, in milligrams per liter;
- D = (DO) deficit = $C_s - C$, in milligrams per liter;
- D_c = critical (maximum) deficit, in milligrams per liter;

D_u = DO deficit in receiving waters upstream of inflow point, in milligrams per liter;
D_0 = initial DO deficit, in milligrams per liter;
E = longitudinal dispersion coefficient, in square feet per second;
f = self-purification ratio, K_2/K_1;
H = stream depth, in feet;
K_1 = deoxygenation constant of carbonaceous BOD, per hour;
K_2 = atmospheric reaeration coefficient, per hour;
k = number of hourly lags;
k = coefficient;
L = Length;
L_0 = remaining carbonaceous BOD concentration, in milligrams per liter;
M = Mass;
Q = streamflow, in cubic feet per second;
Q_c = combined sewer flow, in cubic feet per second;
Q_d = DWF treated effluent, in cubic feet per second;
Q_r = urban runoff carried by the separate storm sewer, in cubic feet per second;
Q_w = wet-weather flow (WWF) treated effluent, in cubic feet per second;
R_d = fraction removal of BOD achieved by the DWF treatment facility;
R_w = fraction removal of BOD achieved by the WWF treatment facility;
R_0 = deficit load ratio = D_0/L_0;
R_t = pollutant removal, as a percentage;
$\bar{R}_t$ = maximum pollutant removal, as a percentage;
$\underline{R}_t$ = minimum pollutant removal, as a percentage;
$r_Q(k)$ = sample estimate of lag-k autocorrelation coefficient for runoff;
S = sources and sinks of the substance C, M/L^3T;
T = Time;
T = stream temperature, in degree Celsius;
t_c = elapsed time at which critical deficit occurs, in hours or days;
U = flow velocity in stream, in feet per second;
x = distance downstream, in feet or miles;
Z_{CO} = annual cost, in dollars per acre of combined sewered area;
Z_{ST} = annual cost, in dollars per acre of storm sewered area; and
Z^* = total annual cost, in dollars per acre;
α_1, α_2 = regression coefficients;
β, β_1, β_2 = regression coefficients; and
γ_1, γ_2 = regression coefficients.